State of California | Business, Transportation and Housing Agency

Memorandum

To: Mr. Robert Finney
Resident Engineer
Benicia-Martinez Office

Date: February 8, 2001

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04-0440U1
Benicia-Martinez Br.
Retrofit Project
Bridge No. 28-0153

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Division of Structural Foundations - MS 5
Office of Structure Foundations

Subject: Benicia-Martinez Bridge Seismic Retrofit Construction-Geologic and Constructability Issues

This report was initiated by the request from Toll Bridge Program Management Office for a general overview of geotechnical conditions encountered during foundation construction for the above mentioned bridge retrofit. This report should be used as a material handout to help contractors to have a better understanding of the site geological conditions and is supplemental to "Critique of the Foundation Retrofit for Piers #4 through #12" prepared by David Nesbitt from the Martinez Construction Office. Prior to bid, the contractor should review Final Foundation Recommendation and Log of Test Borings. It is strongly recommended that each bidder inspect cores.

Characteristics of the Rock Mass

The rock types and their characteristics (rock quality designation, recovery, color, degree of weathering, hardness, texture descriptions, and fracture density) are presented in the Log of Test Borings. During retrofit construction the information given in LOTB,s was generally confirmed. The Contractor (FCI Corp.) did not experience significant difficulties during shaft construction until construction of the new caisson (No.12 and No. 14) at Pier 9. The other two caissons at this location were constructed with no difficulties. Construction problems at Pier 9 were severe and caused delays in project completion. For this reason, Pier 9 geotechnical issues have been given major attention in this report. Based on the pile driving record, a steel caisson was driven to the elevation –132.0 feet and after central relief drilling redriven to specified pile tip elevation -150.0 feet (18 feet into the bedrock). After completion of the pile driving the contractor started rock socket construction. Some caving was reported during drilling but the specified pile tip was reached at the elevation –190 feet. The shaft collapsed soon after completion of drilling. The contractor was using water-based slurry and did not conduct a 24 hours drilling operation. The caving mitigation method is described in a report written by D. Nesbitt.

At Pier 9 two additional boreholes were drilled (west and east side of the pier) for better understanding of the encountered problem.

The review of the logs of test borings indicates that the rock beneath the Pier 9 consists of two distinctive types. At borehole 00-1-9 (east side) the rock was described as interbedded claystone and siltstone containing thin layers of sandstone. This formation (Panoche ?) is characterized by various degrees of fracturing ranging from 0% to 50% RQD. Since this predominantly fine-grained rock has been structurally tilted to 70 degrees, there are zones of close spaced bedding which have been subjected to intense weathering and possible shear displacement. Rock strength of this formation (based on unconfined compressive strength tests) reaches a maximum 1973 psi although for the most shaft length has soil like appearance. At borehole 00-2-9 (west side) rock is much harder and represented predominantly by fine grained sandstone. This formation (Martinez ?) is characterized by the rock being less fractured than on the east side, with RQD ranging from 0% to 100%. Sandstone is described as slightly weathered, moderately hard and massive. Unconfined compressive strengths of this formation vary from 1000 psi to 2486 psi. Sandstone is less fractured with the similar to the claystone angle of dip, which averages 50-70 degrees. The bedrock described in borehole 94-1,drilled 190 feet from the pier location, is an average representation of the both formations. Harder sandstone is interbedded with weaker (intensely weathered) siltstone.

The rock characteristics are well presented on the geophysical data. The highest shear wave and P-wave velocities were recorded for the sandstone at borehole 00-2-9 (3000 ft/sec average). Borehole 00-1-9 has an average P-S velocities for the Panoche Formation (2500 ft/sec) with distinctive weak zones of relatively low velocities (elevation -160.0 feet to -165.0 feet and -170.0 feet to -180.0 feet. Similar low velocities were recorded in borehole 94-1 at the same elevations, although a weak zone appears to extend here to the elevation –170.0 feet.

The zone at the retrofitted bridge location is a wedge caught between the east inclined Southampton thrust and the west dipping Sulfur Springs thrust zone. As a result the geological units appear to be folded, faulted, and thrusted. At Pier 9 there is the possible contact between Panoche and Martinez formations and the bedrock units are intensely weathered, fractured and sheared with shear waive velocities ranging between 1800 to 4000 ft/s.

The discontinuities effected the variation in degree of weathering from decomposed or intensely weathered to fresh (siliceous sandstone). Subsequently the rock hardness varies significantly. For example at the Pier 9 location rock hardness varies from 2486 psi (00-2-9) to values determined by the pocket penetrometer to be close to 50 psi (samples were not suitable for qu testing) at borehole 00-1-9. On some of the rock samples point load index tests were performed (short core fragments). Most of the core samples tested for the unconfined compression strength failed in shear (along incipient joints). In situ rock has higher qu values because of confinement. Hard rock caused some drilling problems at Pier 12 (pile load test). The contractor was forced to use diamond tip nubs. The point load test performed on some sandstone fragments recovered during drilling at this location confirmed qu values of being close to 7000 psi (oral information from contractor).

Typically for this job site, bedrock characteristics vary by the rock type and by layer. In some borings, good quality rock is described beneath a highly weathered and decomposed material. The intensely fractured siltstone may occur between fresh and slightly fractured sandstone beds. The rock characteristics vary along the dip (50 to 70 degrees) or along strike (N40-50 W), rather than being uniform throughout a vertical thickness below the rock surface. The experience from construction at Pier 12 (pile load Test) confirmed that weaker rock might be located below fresher, harder material. Because of high dipping angles of the bedding planes the location of various fractured and weathered rock layers may change within a short lateral distance.

The joints in both formations represent significant flow paths for any water (ground water or water introduced during construction). Significant loss of circulation was reported during construction of the shaft at Pier 12 (pile load test).

Some claystone and siltstone (shale) are characterized as a "weak rock". Shale has a finely laminated structure, which imparts fissility approximately parallel to bedding. Shale (shaley siltstone and shaley claystone) is usually intensely weathered and fractured and possess a significant content of clay minerals. It has the appearance of a rock but behaves partially as a soil. Shale exhibits potential to swell and slake when exposed to water. Slaking of shale may occur within an hour of exposure to water. This was proved by a slake test in the laboratory. Slaking material may undermine steeply deeping overlaying, fractured rock, which fails as a block. During construction of the shaft at Pier 12 (pile load shaft), the contractor experienced a major cave in which resulted in adding approximately 20 cubic yards of concrete to the estimated volume. The caliper test performed at the shaft at Pier 8 (retrofit) confirmed significant oversizing at this borehole as well. Only the bottom 10 feet, drilled 24 hours before, was close to the intended socket size (1676 mm-66 inches). The upper portion of the shaft, constructed over the previous four days had a much lager diameter. The estimated diameter was based on the placed concrete volume and was as high as 2844 mm (112 inches) at one location. Both shafts were rejected after gamma and cross-sonic testing. The contractor was using water-based slurry and no salt was added. The shear strength of the rock when exposed to water during steel pile installation will be greatly reduced because the sidewall of the shaft may develop a layer of slaked mud or clay. The State is currently sponsoring Benicia-Martinez rocks slake test study with different type of polymers and salt additives. The result of the study will be available to the contractors.

If you have any questions regarding presented comments, please call at (916) 227-7047.

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Associate Engineering Geologist

Attachments:
1. Pier 9 Log of Test Borings
2. Pier 4-12 Pile Driving Record

Cc: RFox -TBGS
EWiecha -TBP